APPENDIX D

DESIGN EXAMPLES—BUILDING SYSTEMS

D-1. Introduction. This appendix gives illustrative examples for designing various types of lateral systems. Generally, the calculations determine earthquake lateral forces and their distribution to the resisting elements of the buildings; some examples covering frames, walls, diaphragms, and foundations are essentially complete. Calculations are not given where ordinarily accepted design procedures are involved, such as sizing and detailing members once forces are determined.

D-2. Use of this appendix. This appendix is purely advisory; it is not intended to place super-restrictions on the manual. This appendix is not a handbook for the inexperienced designer. Neither the manual, nor the manual supplemented by this appendix, can replace good engineering judgment in specific situations. Designers are urged to study the entire manual.

D-3. Commentary.

a. Unless otherwise indicated, all design examples in this appendix are based on Zone 4, where Z = 0.40. But the principles and methods for determining lateral forces are alike for all zones.

b. Examples D-1, D-2, D-3, and D-5 are for the same basic building, using (1) bearing walls, (2) concrete frames, (3) steel frames, and (4) frames in combination with shear walls (a dual bracing system), respectively. These examples tend to illustrate the relationship between architectural features (fenestration and materials of construction) and structural design.

c. A 10-pound-per-square-foot weight is added to the roof for the seismic effect of the upper half of the top-story partitions.

d. It is assumed that stairs are detailed so as not to transmit shears from floor to floor. Also, removable and special partitions (such as utility room walls) will be made flexible or isolated so as not to affect the distribution of lateral loads or to act as shear walls.

e. Metal-deck roofs are considered to form flexible diaphragms, and roof loads are distributed according to tributary area rather than relative rigidity of walls below.

D-4. Design examples.

Fig. No.	*Description of Design Examples*
D-1	*Box System.* A two-story building with bearing walls in concrete using a series of interior, vertical load-carrying columns and girder bents.
D-2	*Concrete Ductile Moment Resisting Space Frame.* A three-story building with a complete ductile moment resisting space frame in concrete without shear walls.
D-3	*Steel Ductile Moment Resisting Space Frame and Steel Braced Frame.* A three-story building with transverse special moment resisting frames and longitudinal frames with chevron bracing.
D-4	*Dual Bracing System.* A two-story building in concrete with a ductile moment resisting space frame and with shear walls.
D-5	*Dual Bracing System.* A three-story building with a ductile moment resisting space frame in structural steel and with shear walls in concrete.
D-6	*Wood Box System.* A two-story wood framed building, using wood floor and roof decks, and wood stud walls with plywood sheathing.
D-7	*Special Configuration.* A one-story building with concrete bearing walls on three sides and open on one side.
D-8	*L-Shaped Building.* A three-story building with bearing walls in concrete, using a series of interior vertical load-carrying columns and girder bents.

DESIGN EXAMPLE D-1

Building With A Bearing Wall:

Description of Structure. A two-story administration building with bearing walls in concrete, using a series of interior, vertical load-carrying column and girder bents. The structural concept is illustrated on Sheets 3 and 4.

Construction Outline.

Roof:
- Built-up, 5-ply.
- Metal decking with insulation board.
- Suspended ceiling.

2nd Floor:
- Metal decking with concrete fill.
- Asphalt tile.
- Suspended ceiling.

1st Floor:
- Concrete slab-on-grade.

Exterior Walls:
- Bearing walls in concrete, furred with GWB finish

Partitions:
- Non-structural removable dry—wall, except concrete as structurally required.

Design Concept. Since the structure is without a complete load-carrying space frame, the Rw-factor is 6. The metal deck roof system forms a flexible diaphragm, therefore the roof loads are distributed to the shear walls by tributary area rather than by second story wall stiffnesses. The roof diaphragm being flexible will not transmit accidental torsion to the shear walls. The metal deck with concrete fill system for the second floor forms a rigid diaphragm. The shear walls react to the forces from the diaphragm, therefore the relative rigidities of the various walls and the individual piers must be determined. This is necessary so that a logical and consistent distribution of story shears to each wall and pier can be made. The wall analysis utilizes the Design Curve for Masonry and Concrete Shear Walls on Figure 6-4.

Discussion. A 10 psf partition load is included in the seismic roof loading but is not included in the vertical design. The stairs are isolated so that they will not transmit shears from floor to floor. The walls along Lines Ⓐ Ⓒ ③ & ⑤ act as vertical cantilever beams joined by struts at the floor lines. The overturning moments are distributed to the individual piers in proportion to the pier stiffnesses. The end wall along Line ⑦ abuts an existing building, therefore a wall with no openings is provided. The spandrels in wall along Line ① must be designed to transfer vertical shears due to shear wall action.

Figure D-1. Box system.

Loads.

Roof:

5-ply roofing	=	6.0 p.s.f.
1” Insulation	=	1.5
Steel deck	=	2.3
Steel purlins	=	3.7
Steel girders & columns	=	1.2
Ceiling	=	10.0
Miscellaneous	=	1.0
Dead Load		25.7 p.s.f.
Add for seismic: Partitions		10.0 p.s.f.
Total for seismic		35.7 p.s.f.*
Live Load		20 p.s.f. (no snow)

2nd Floor:

Finish	=	1.0 p.s.f.
Steel deck	=	3.1
Concrete fill	=	32.0
Steel beams	=	5.9
Steel girders & columns	=	1.5
Partition	=	20.0
Ceiling	=	10.0
Miscellaneous	=	1.0
Dead Load	=	74.5 p.s.f.*
Live Load	=	50.0 p.s.f.

Materials.

Structural steel	F_y	=	36 k.s.i.
Concrete	f'_c	=	4,000 p.s.i., $E_c = 3.6 \times 10^6$ psi
Reinforcing steel	f_y	=	40,000 p.s.i.
Allowable soil pressure .		=	3,000 p.s.f. Vertical Load
Allowable soil pressure ...		=	4,000 p.s.f. Vertical plus Seismic

* Weight of shear walls are not included here. The weight of the concrete shear walls are calculated on pages 4 and 5. The weights of the exterior windows and architectural wall panels are included in the partition weights.

Figure D-1. Continued

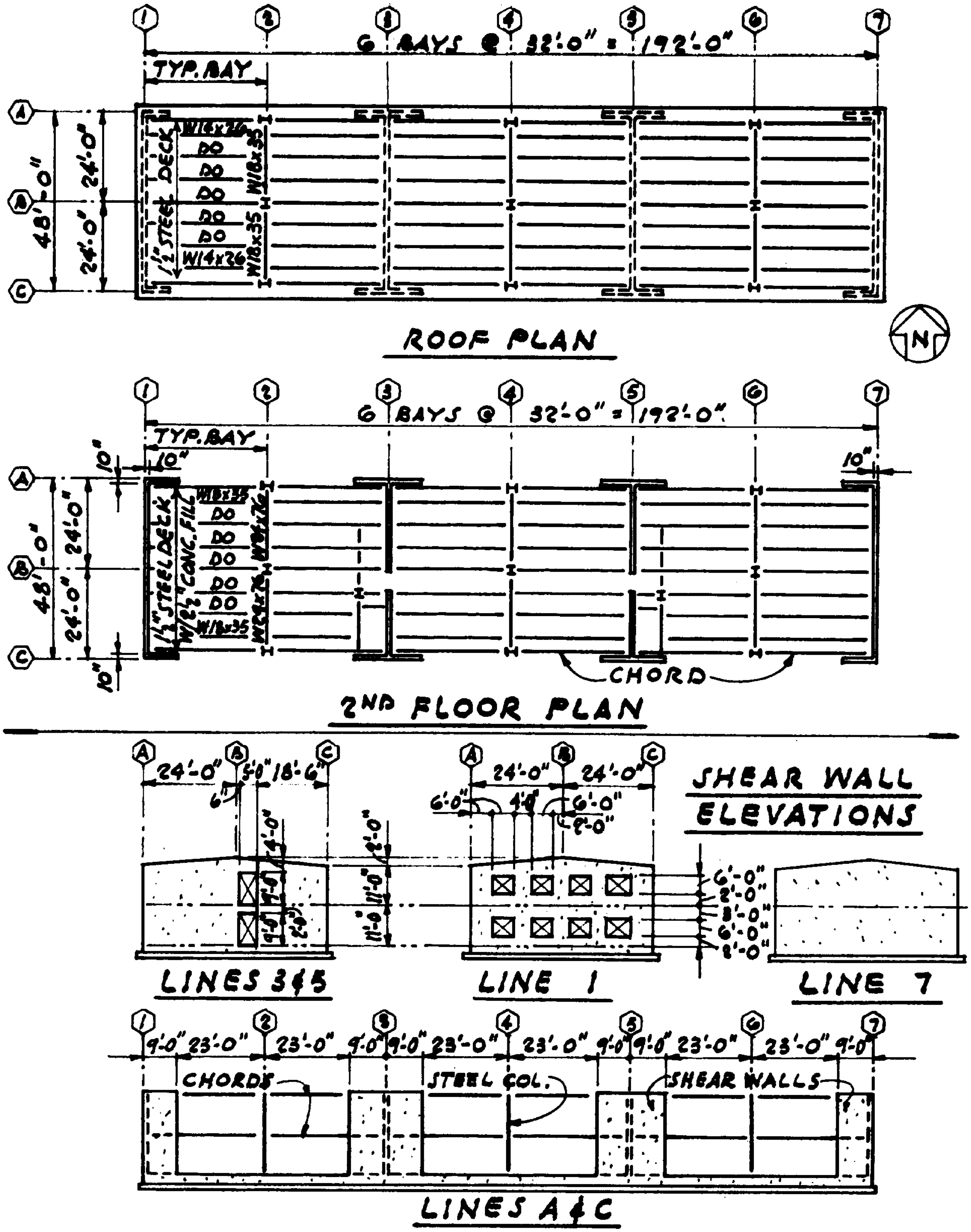
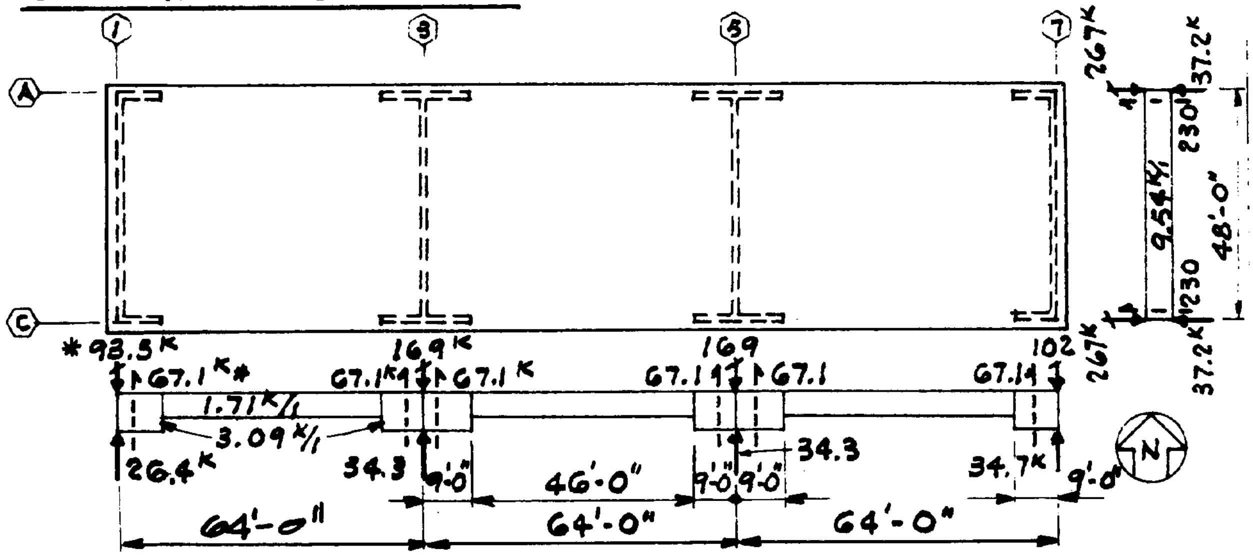


Figure D-1. Continued

DISTRIBUTION OF BUILDING WEIGHT TO ROOF DIAPHRAGM @ 100 % G



10" CONC. WALLS 0.125 KSF

SIDEWALLS A & C: 5.5' x 0.125 = 0.688 K/l, x 2 = 1.38 K/l

CROSSWALLS 1,3,5 & 7: 6.0' x 0.125 = 0.75 K/l, x (2 x .91 + .76 + 1.0) = 2.69 K/l

WALL ON 1 : .76 x 0.75 x 46.33' = 26.4 (76 % SOLID)

WALL ON 7 : 1.0 x 0.75 x 46.33' = 34.7 (100 % SOLID)

WALL ON 3 & 5 : .91 x 0.75 x 46.33' = 31.6 (91 % SOLID)

WALLS ON A & C:

9' WALLS	0.688 x 9' =	6.2 K	x 2 = 12.4 K
18' WALLS	=	12.4 K	x 2 = 24.8 K
			37.2 K

(P.2)

		E-W LOADS	N-S LOADS
ROOF	0.0357 KSF	x 192' = 6.85 K/l	x 48' = 1.71 K/l
WALLS		2.69	1.38
		9.54 K/l	3.09 K/l

*SAMPLE CALCULATION OF WALL $\boxed{1}$

DISTRIBUTION OF DIAPH. WT TO WALL $\boxed{1}$ = 1.71 K/l x 64'/2 = 54.7 } 67.1 K

" " SIDEWALL " " " = 1.38 x 9' = 12.4 }

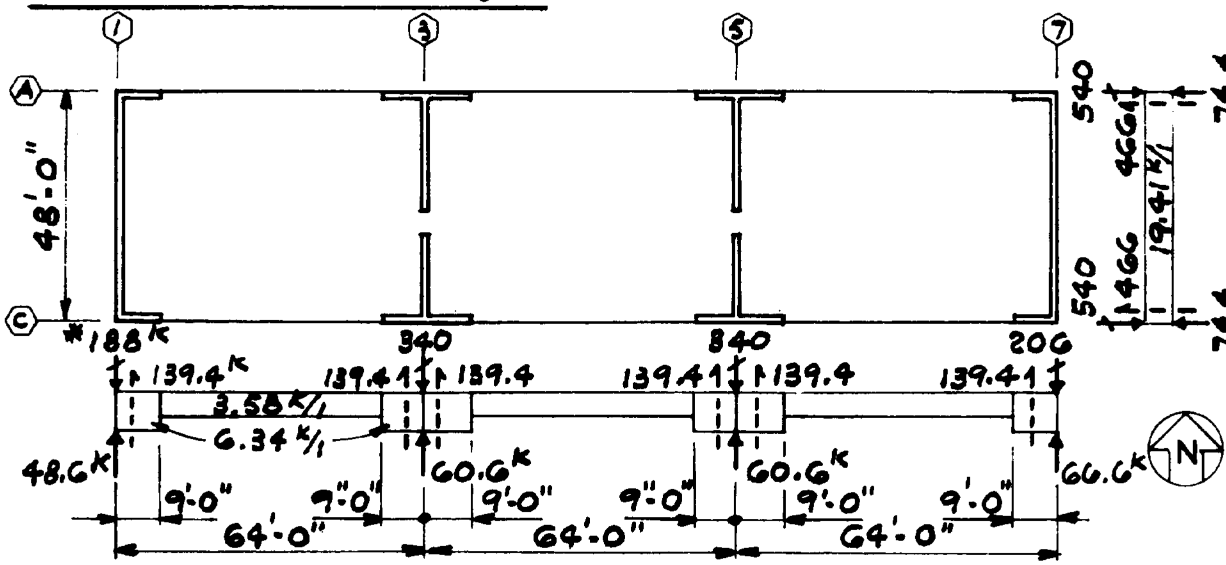
THE WT CONTRIBUTION FROM IN-PLANE SHEAR WALL $\boxed{1}$ (SEE ABOVE) 26.4

THE TOTAL WT CONTRIBUTION TO SHEAR WALL $\boxed{1}$ 93.5 K

TOTAL TRIB. WT OF ROOF & ALL WALLS W_R = 584 K @ 100 % G

Figure D-1. Continued.

DISTRIBUTION OF BUILDING WEIGHT AT 2ND FLOOR DIAPHRAGM @ 100 % G



10" CONC. WALLS = 0.125 KSF

WALL ON 1 : $.73 \times 0.125 \times 11.5' \times 46.33 = 48.6^K$

WALL ON 7 : $1.0 \times 0.125 \times 11.5 \times 46.33 = 66.6^K$

WALLS ON 3 & 5: $.91 \times 0.125 \times 11.5 \times 46.33 = 60.6^K$

WALLS ON A & C:

9' WALLS $0.125 \times 11' \times 9' = 12.4^K \times 2 = 24.8^K$

18' WALLS $= 24.8^K \times 2 = 49.6^K$

74.4^K

	(P.2) E-W LOADS	N-S LOADS
FLOOR	$0.0745\ KSF \times 192' = 14.30\ K/l$	$\times 48' = 3.58\ K/l$
WALL	$0.73 \times 0.125\ KSF \times 11.5' = 1.05$	$2 \times 0.125 \times 11' = 2.76$
	$1.0 \times 0.125\ KSF \times 11.5' = 1.44$	$6.34\ K/l$
	$2 \times .91 \times 0.125\ KSF \times 11.5' = 2.62$	
	$19.41\ K/l$	

*SAMPLE CALCULATION FOR WALL $\boxed{1}$

DISTRIBUTION OF DIAPH. WT TO WALL $\boxed{1}$ $= 3.58\ K/l \times 64/2 = 114.6$ } $= 139.4^K$

" " SIDEWALL " " " $= 2.76 \times 9' = 24.8$

THE WT CONTRIBUTION FROM THE IN-PLANE SHEAR WALL $\boxed{1}$ (SEE ABOVE) $= 48.6$

THE TOTAL WT CONTRIBUTION TO SHEAR WALL $\boxed{1}$ $= 188.0^K$

TOTAL TRIB WT OF 2ND FLR DIAPH & 2ND FLR TRIB WALLS $W_P = 1080^K$ @ 100% G

Figure D-1. Continued

LATERAL FORCES (BOTH DIRECTIONS)

$V = \frac{ZIC}{R_w} W$ SEAOC FORMULA 1-1

$Z = 0.40$ (ZONE 4) TABLE 1-A

$I = 1.0$ TABLE 1-C

$R_w = 6$ (BEARING WALL SYSTEM CONCRETE SHEAR WALLS) TABLE 1-G

$S = 1.5$ (ASSUME SOIL S_3) TABLE 1-B

$h_n = 22$ FT. (AVG.)

$C_t = 0.020$

$T = C_t (h_n)^{3/4} = 0.020\,(22)^{3/4} = 0.203$ FORMULA 1-3

$C = 1.25S/(T)^{2/3}$ FORMULA 1-2

$= 1.25 \times 1.5 \div (0.203)^{2/3} = 5.43$

BUT NEED NOT EXCEED 2.75

$V = \frac{0.40 \times 1.0 \times 2.75}{6} W = 0.183W$

$= 0.183 \times 1614 = 295^K$, SAY 300^K

LEVEL	h FT.	Δh FT.	w K	Σw K	wh K·FT.	wh/Σwh	F_x K	V K	ΔM_{OT} K-FT.	M_{OT} K-FT.
R	22		534		11,748	.50	150			
		11		534				150	1650	1650
2	11		1080		11,880	.50	150			
		11		1614				300	3300	4950
GRD	0									
Σ			1614		23,628	1.00	300		4950	

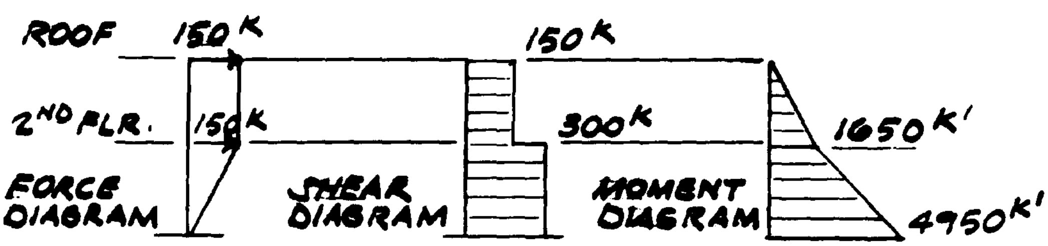
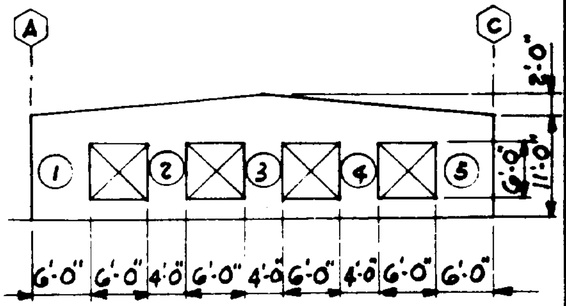
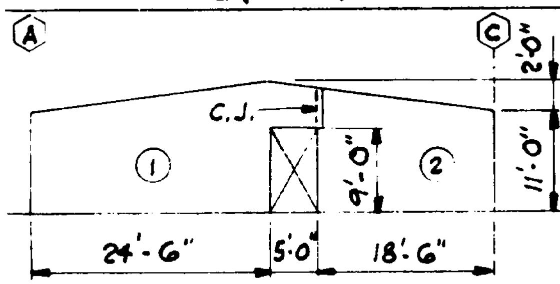


Figure D-1. Continued.

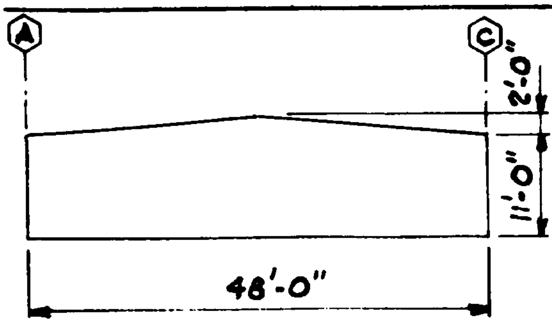
RELATIVE RIGIDITIES
2ND STORY WALLS

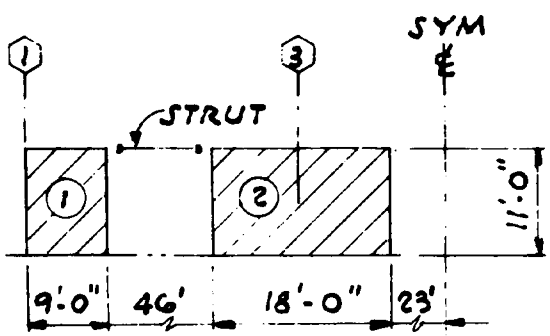
WALLS	PIER	H	D	H/D	Δ FIG. 6-4	R	ΣR WALL
	1 & 5 (CORNER FIXED)	6'	6	1.0	0.09	11.1 × 2 PIERS = 22.2	
	2, 3, 4 (RECT. FIXED)	6'	4'	1.5	0.22	4.5 × 3 PIERS = 13.5	
WALL 1 - 2ND STORY					Δ(1-5) = 1/R = 0.028	ΣR = 35.7	
	SOLID WALL (CORNER CANT)	12' AVE	48'	0.25	0.018		
Δ(WALL) = 0.018 - 0.008 + 0.028 = 0.038	SUBTRACT BAND @ WINDOW (CORNER CANT)	6'	48'	0.125	<0.008>		
R(WALL) = $\frac{1}{\Delta(WALL)}$ = 26.3					ΣΔ(WALL) = 0.038		ΣR = $\frac{1}{\Sigma\Delta W}$ = 26.3
	1 (CORNER CANT.)	12' AVE	24.5	0.49	0.044	22.7	
WALL 3 - 2ND STORY (WALL 5 SIM.)	2 (CORNER CANT.)	12'	18.5	0.65	0.065	15.4	
FOR THIS EXAMPLE, CONTROL JT. IS PROVIDED TO MAKE WALL MORE FLEXIBLE, THEREBY DISTRIBUTING MORE LOAD TO WALL 7						Σ 38.1	ΣR = 38.1

NOTE: SINCE ALL WALLS ARE THE SAME THICKNESS (i.e. 10") THE VALUES FROM FIG. 6-4 FOR 12" WALLS MAY BE USED FOR RELATIVE RIGIDITIES WITHOUT ADJUSTMENT.

Figure D-1. Continued.

RELATIVE RIGIDITIES
2ND STORY



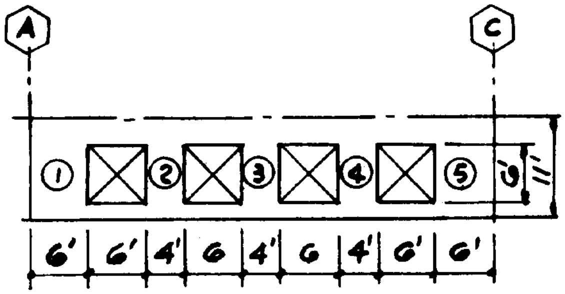


WALL	PIER	H	D	H/D	△ FIG.6-4	R	ΣR WALL
WALL 7 - 2ND STORY	1 CORNER CANT	12 AVE	48	0.25	0.018	55.5	55.5
WALL C - 2ND STORY (WALL A - SIM.)	1, 4 CORNER CANT	11	9	1.22	0.22	4.5 × 2 PIERS 9.1	
	2, 3 RECT. CANT	11	18	0.61	0.075	13.3 × 2 PIERS 26.6	ΣR = 35.6

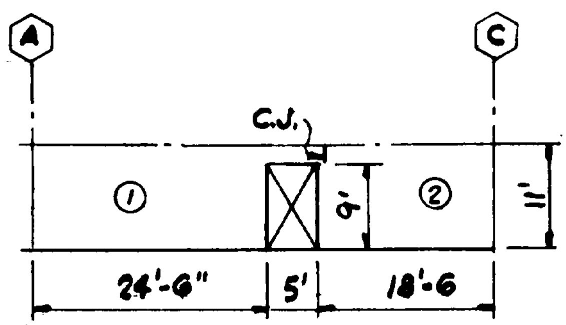
Figure D-1. Continued.

RELATIVE RIGIDITIES

1ST. STORY WALLS



WALL 1 1ST STORY



WALL 3 - 1ST STORY
(WALL 5 - SIM.)

WALL	PIER	H	D	H/D	Δ FIG.6-4	R	ΣR WALL
WALL 1 1ST STORY	1 & 5 (CORNER CANT.)	6	6	1.0	0.09	11.1 × 2 PIERS = 22.2	
	2,3,4	6	4	1.5	0.22	4.5 × 3 PIERS = 13.5 ; ΣR = 35.7	
					Δ(1-5) = $\frac{1}{R}$ = 0.028		
	SOLID WALL (CORNER CANT.)	11'	48'	.23	0.017		
$\Delta(WALL) = 0.017 - 0.008 + 0.028 = 0.037$	SUBTRACT BAND @ 6' WINDOW (CORNER CANT.)	6'	48'	0.125	⟨0.008⟩		
$\Sigma R_{WALL} = \frac{1}{0.037} = 27$					ΣΔ (WALL) = .037		$\Sigma R = \frac{1}{\Sigma \Delta W} = 27$
WALL 3 - 1ST STORY (WALL 5 - SIM.)	1 (CORNER CANT)	11'	24.5	0.45	0.037	27.0	
	2 (CORNER CANT.)	11'	18.5	0.59	0.056	17.9 ; 44.9	ΣR = 44.9

Figure D-1. Continued.

RELATIVE RIGIDITIES

1ST STORY WALLS

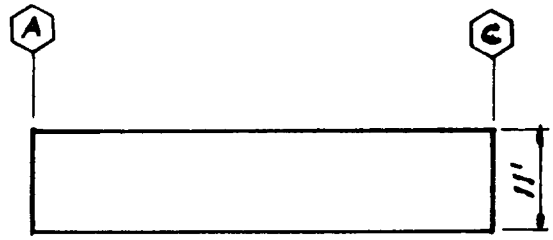
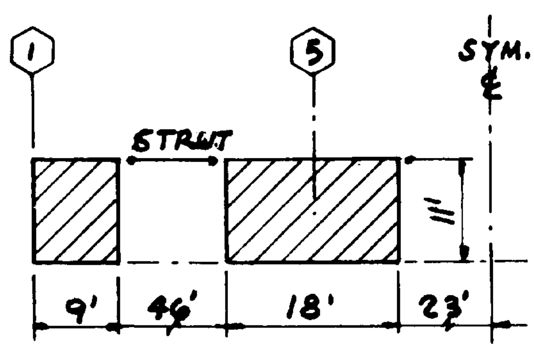
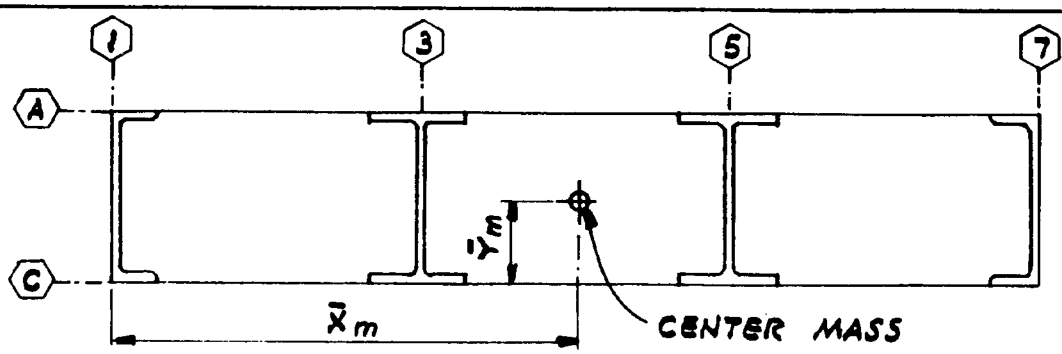
WALL	PIER	H	D	H/D	Δ FIG. 6-4	R	ΣR WALL
 WALL 7 1ST. STORY	1 (CORNER CANT.)	11'	48'	0.23	.017	58.8	58.8
 WALL C - 1ST STORY (WALL A SIM.)	1,4 (CORNER CANT.)	11'	9'	1.22	0.22	4.5× 2PIERS 9.0	
	2,3 (RECT. CANT.)	11'	18'	0.61	0.075	13.3× 2PIERS 26.6	ΣR= 35.6

Figure D-1. Continued.

CENTER OF MASS AND CENTER OF RIGIDITY
ROOF DIAPHRAGM



			CENTER OF MASS			CENTER OF RIGIDITY		
	X	Y	W (P.4)	W • Xm	W • Ym	RIGIDITY R	R • Xr	R • Yr
WALL 1	0.42'		93.5	39				
WALL 3	64		169	10816				
WALL 5	128		"	21632				
WALL 7	191.58		102	19541				
			533.5	52028				
WALL A		47.58	267		12704			
WALL C		0.42	"		112			
			534		12816			

CENTER OF MASS OF ROOF DIAPHRAGM;

$$\bar{X}_m = \frac{52028}{533.5} = 97.5 \qquad \bar{Y}_m = \frac{12816}{534} = 24'$$

CENTER OF RIGIDITY:

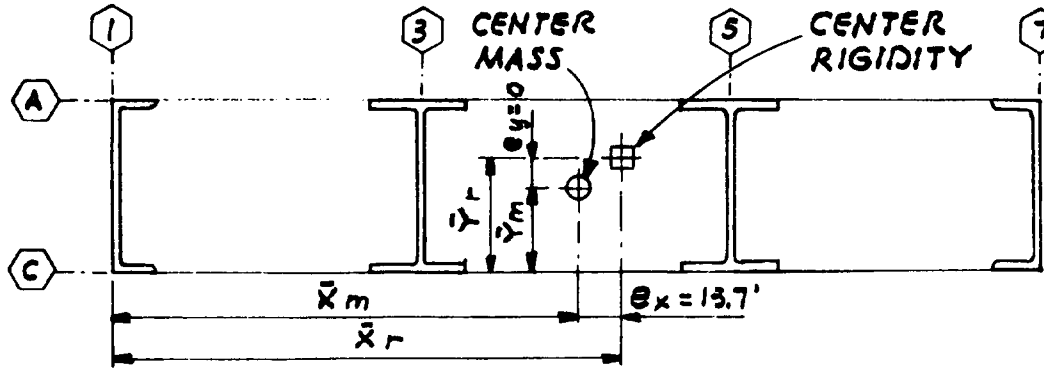
CALCULATIONS NOT REQUIRED SINCE ROOF DIAPHRAGM IS FLEXIBLE AND SEISMIC FORCES ARE DISTRIBUTED BY TRIBUTARY AREA.

CENTER OF MASS OF ROOF DIAPHRAGM IS REQUIRED SINCE THE ECCENTRICITY OF THIS MASS EFFECTS THE TORSIONAL FORCE ON THE RIGID 2ND FLOOR DIAPHRAGM BELOW.

Figure D-1. Continued.

CENTER OF MASS AND CENTER OF RIGIDITY

2 ND FLOOR DIAPHRAGM



	X	Y	CENTER OF MASS			CENTER OF RIGIDITY		
			W (P.5)	W·Xm	W·Ym	RIGIDITY R (P.10)	R·Xr	R·Yr
WALL 1	0.42		188	79		27	11	
WALL 3	64		340	21760		44.9	2874	
WALL 5	128		"	43520		"	5747	
WALL 7	191.58		206	39465		58.8	11265	
			*1074	104824		175.6	19897	
WALL A		47.58	540		25693	35.6	NONE	1694
WALL C		0.42	"		227	"	NONE	15
			*1080		25920	71.2		1709

* TOTAL WTS. DO NOT CORRESPOND DUE TO ROUNDING OFF.

CENTER MASS: $\bar{X}_m = \frac{104824}{1074} = 97.6'$ CENTER RIGIDITY: $\bar{X}_r = \frac{19897}{175.6} = 113.3'$

$\bar{Y}_m = \frac{25920}{1080} = 24'$ $\bar{Y}_r = \frac{1709}{71.2} = 24'$

ECCENTRICITY OF ROOF MASS W/RESPECT TO 2ND FLR. CENTER RIGIDITY

$e_x = 113.3 - 97.5 = 15.8'$ $e_y = 24' - 24' = 0$

(P.11)

ECCENTRICITY OF 2ND FLR MASS W/RESPECT TO 2ND FLR. CENTER RIGIDITY

$e_x = 113.3' - 97.6' = 15.7'$ $e_y = 24' - 24' = 0$

Figure D-1. Continued.

DISTRIBUTION OF SEISMIC FORCES

FROM ROOF DIAPHRAGM TO WALLS BELOW

THE ROOF DIAPHRAGM IS FLEXIBLE; THEREFORE THE FORCES ARE OBTAINED BY THE TRIBUTARY AREA METHOD. THE 100%g FORCES OF P.4 ARE SCALED IN PROPORTION TO THE 150^K ROOF FORCE (P.6).

WALL	100%g	$F_R = 150$
1	94	26.4
3	169	47.5
5	169	47.5
7	102	28.6
	534^K	150^K

WALL	100%g	$F_R = 150$
A	267	75
C	267	75

FROM SECOND FLOOR DIAPHRAGM TO WALLS BELOW

THE SECOND FLOOR DIAPHRAGM IS RIGID; THEREFORE IT REDISTRIBUTES FORCES FROM THE SECOND FLOOR AND ABOVE.

N-S DIRECTION

P.6 P.12 P.6 P.12

$$M_T = \Sigma F_x e_x = \underset{ROOF}{150^K} \times 15.8' + \underset{2ND}{150} \times 15.7 = 4725^{K'}$$

$$M_A = 300^K \times (0.05 \times 192') = \quad 2880^{K'}$$

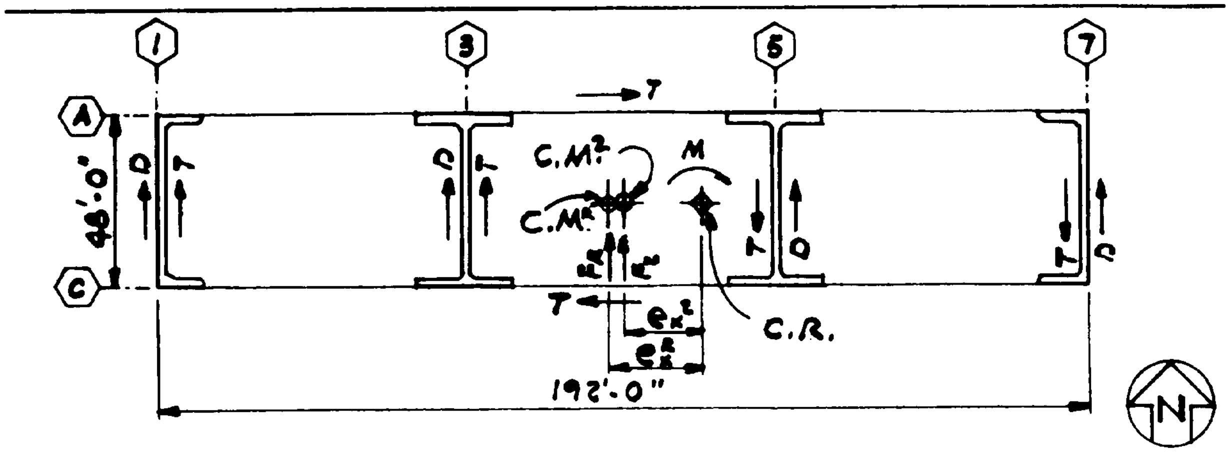
E-W DIRECTION

$$M_T = \quad 0$$

$$M_A = 300 \times (0.05 \times 48) = \quad 720^{K'}$$

Figure D-1. Continued.

DIAGRAM SHOWING DIRECT SHEAR AND TORSIONAL SHEAR FORCES IN PLAN



NOTES:

C.R. = CENTER OF RIGIDITY
C.M. = CENTER OF MASS
T = TORSIONAL SHEAR FORCE
D = DIRECT SHEAR FORCE
F_R = FORCE FROM ROOF DIAPH (N-S)
F_2 = FORCE FROM 2ND DIAPH (N-S)
e_x^R = ECCENTRICITY OF ROOF MASS W/ RESPECT TO 2ND FLR C.R
e_x^2 = " " 2ND FLR " " " " " "
M_T = TORSIONAL MOMENT
M_A = ACCIDENTAL TORSION
$M = M_T + M_A$

$$V_D = \frac{K}{\Sigma K} V$$

$$V_T = \frac{Kd^2}{\Sigma Kd^2} \cdot \frac{M_T}{d} = \frac{Kd}{\Sigma Kd^2} \cdot M_T$$

$$V_A = \frac{Kd^2}{\Sigma Kd^2} \cdot \frac{M_A}{d} = \frac{Kd}{\Sigma Kd^2} \cdot M_A$$

Figure D-1. Continued.

DISTRIBUTION FROM SECOND FLOOR DIAPHRAGM TO WALLS BELOW — CONTINUED

N—S DIRECTION

WALL	K	$\frac{K}{\Sigma K}$	V_D	d	d^2	Kd^2	$\frac{kd}{\Sigma Kd^2}$	V_T	V_A	V_W
1	27.0	.154	46.2	112.9	12,746	344,142	.00353	16.7	10.2	73.1
3	44.9	.256	76.8	49.3	2,430	109,107	.00256	12.1	7.4	96.3
5	44.9	.256	76.8	14.7	216	9,698	.00076	− 3.6	2.2	75.4
7	58.8	.334	100.2	78.3	6,131	360,503	.00533	−25.2	15.4	90.4
	175.6		300							
A	35.6	.500	—	23.6	557	19,829	.00097	4.6	2.8	7.4
C	35.6	.500	—	23.6	557	19,829	.00097	4.6	2.8	7.4
	71.2					863,108				

E—W DIRECTION

WALL	K	$\frac{K}{\Sigma K}$	V_D	d	d^2	Kd^2	$\frac{kd}{\Sigma Kd^2}$	V_T	V_A	V_W
1			—					—	2.5	2.5
3			—					—	1.8	1.8
5			—					—	0.5	0.5
7			—					—	3.8	3.8
A			150					—	0.7	150.7
C			150					—	0.7	150.7
			300							

Figure D-1. Continued.

DISTRIBUTION OF SEISMIC FORCES & OVERTURNING MOMENTS

NORTH-SOUTH DIRECTION

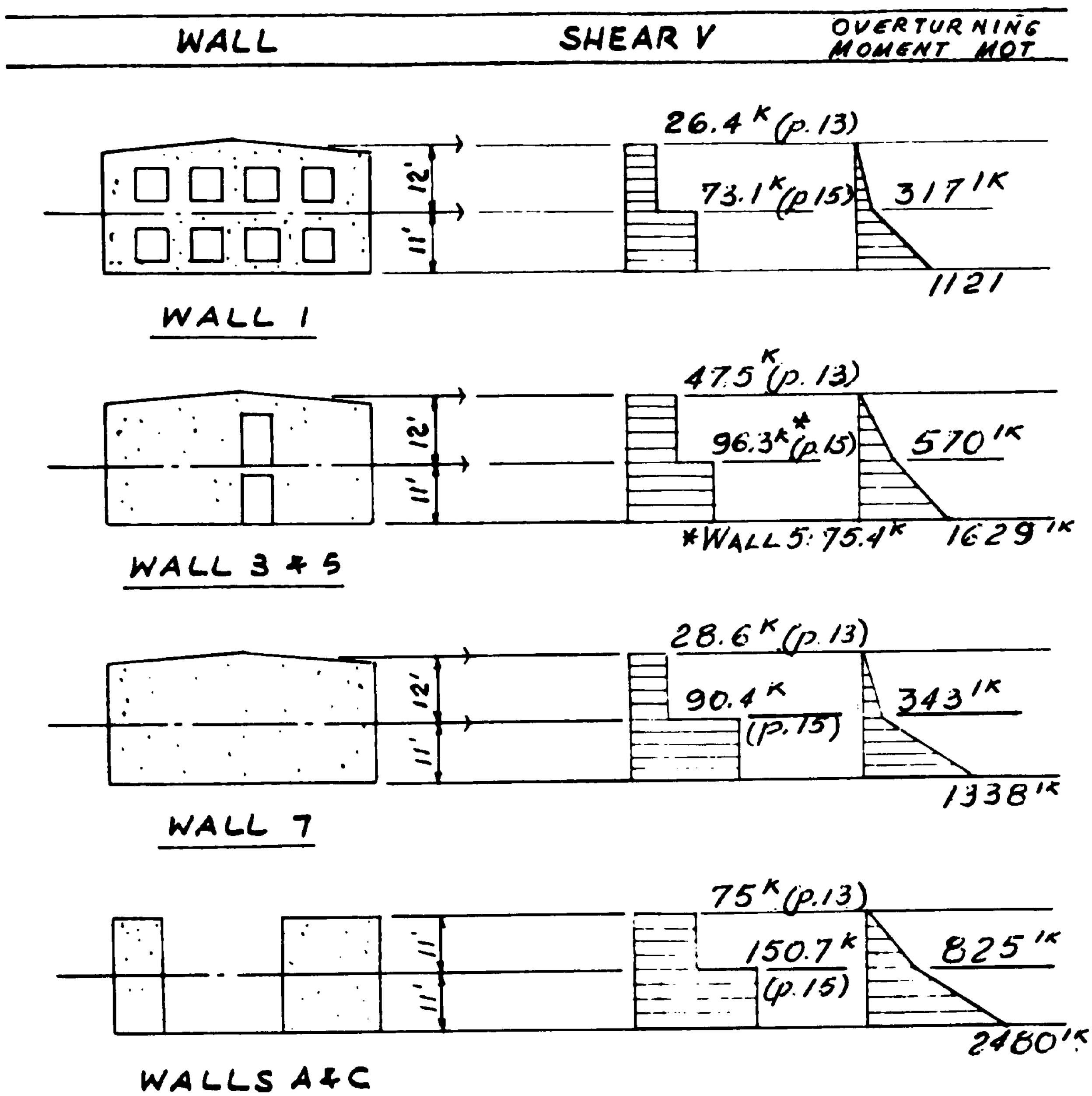


Figure D-1. Continued.

VERTICAL LOAD DESIGN

THESE CALCULATIONS ARE EXTRACTED FROM THE VERTICAL LOAD CALCULATION WHICH ARE REQUIRED FOR COMBINING WITH LATERAL LOADS.

WALL	DEAD LOAD	LIVE LOAD
1	ROOF (p.2) 24.5 #/□' x 16' = 391 #/' (LESS GIRDER + COL.)	ROOF 20# x 16' = 320 #/'
	WALL 125 #/□' x 12' AV'G = 1500	
	LESS WALL OPEN'G	
	$4 \times 6' \times 6' \times \frac{125}{48'}$ #/□' = <-375>	
	ABOVE 2ND — 1516 #/'	
	2ND FLR. 73 #/□' x 16' = 1168 (LESS GIRDER + COL.)	2ND FLR. 50# x 16' = 800
	WALL 125 #/□' x 11 = 1375	
	LESS OPEN'G = <-375>	
	3684 #/'	
	FDN. WALL 125# x 1.5' = 188	
	FTG. (ASSUME 2.5 WIDE) = 563	
	TOTAL DEAD = 4435 #/'	TOTAL LIVE = 1120 #/'

$$\text{FOOTING WIDTH REQ'D} = \frac{4435 + 1120}{3000 \text{ PSF}} = 1.85$$

TRY 2'-6" x 18" CONT. FTG.

NOTE: FOOTING WIDTH TO BE CHECKED FOR SEISMIC LOAD

Figure D-1. Continued.

VERTICAL LOAD DESIGN

WALL	DEAD LOAD	LIVE LOAD
7	ROOF (p.2) 24.5#/□'×16' = 391#/' (LESS GIRDER + COL.) WALL 125#/□'×12' AVG. = 1500#/' 1891#/' 2ND FLR 73#/□'×16' = 1168#/' (LESS GIRDER + COL.) WALL 125#/□'×11' = 1375#/' FDN WALL 125#/□'×1.5' = 188#/' FTG. (ASSUMED 2.75' WIDE = 619#/' TOTAL DEAD = 5241#/'	ROOF 20#×16' = 320#/' 2ND FLR. 50#×16' = 800#/' TOTAL LIVE = 1120#/'

$$\text{FOOTING WIDTH REQ'D} = \frac{5241 + 1120}{3000\text{ PSF}} = 2.12'$$

TRY, 2'-9"× 18" CONT. FTG.

Figure D-1. Continued.

VERTICAL LOAD DESIGN

WALL	DEAD LOAD	LIVE LOAD
3 (WALL 5 SIM)	(p.2) ROOF 24.5#/□' x 32' = 784#/'	ROOF 20#/□' x 32' = 640#/'
	WALL 125#/□' x 12' AVG. = 1500	
	LESS WALL OP'N'G 5' x 9' x 125/48' = <−117>	
	2167	
	(p. 2) 2ND FLR 73#/□' x 32' = 2336	2ND FLR 50#/□' x 32' = 1600#/'
	WALL 125#/□' x 11' = 1375	
	LESS OPEN'G = <−117>	
	5761#/'	
	FDN WALL 125#/□' x 1.5' = 188#/'	
	FTG. (ASSUME 3' WIDE) = 675#/'	
	TOTAL DEAD = 6624#/'	TOTAL LIVE = 2240#/'

FOOTING WIDTH REQ'D = $\frac{6624 + 2240}{3000\,psf}$ = 2.96'

TRY 3'x0" x 18" CONT. FTG.

Figure D-1. Continued.

VERTICAL LOAD DESIGN

WALL	DEAD LOAD	LIVE LOAD
	(p. 2)	
A	ROOF 24.5#/□' x 16' x 3' = 1176#	ROOF 20#/□' x 16' x 3' = 960#
(9' PIER)	WALL 125#/□' x 9' x 11' = 12375#	
(WALL C SIM.)	13551#	
	WT/FT = $\frac{13551}{9}$ = 1506#/'	
	2ND FLR 73.0#/□' x 16' x 3' = 3504#	2ND FLR 50#/□' x 16' x 3' = 2400#
	WALL = 12375#	WT/FT = $\frac{2400}{9'}$ = 270#/'
	WT/FT = $\frac{29430}{9'}$ = 3270#/' 29430#	
	TOTAL DEAD (EXCL. FTG.) = 29430#	TOTAL LIVE = 3360#
	ALLOW SOIL PRESS. = 3000 psf − (300 psf WT FTG) = 2700 psf	
	AREA REQ'D = $\frac{29430+3360}{2700}$ = 12.1□'	TRY, 8'-0" x 20'-0" FTG. REQ'D FOR SEISMIC OVERTURNING. A = 160□'
18' PIER	ROOF 24.5#/□' x 32' x 3' = 2352#	ROOF 20#/□' x 32' x 3' 1920#
	WALL 125#/□' x 18' x 11' = 24750#	
	27102#	
	WT/FT = $\frac{27102}{18}$ = 1506#/'	
	2ND FLR 73#/□' x 32' x 3' = 7008#	2ND FLR 50#/□' x 32' x 3' = 4800#
	WALL = 24750#	
	58860#	
	WT/FT = $\frac{58860}{18}$ = 3270#/'	
	TOTAL DEAD (EXCL. FTG) = 58860#	TOTAL LIVE = 6720#
	ALLOW SOIL PRESS. = 2700 PSF	TRY, 8'-0" x 31'-0" FTG. REQ'D FOR SEISMIC OVERTURNING A = 248□'
	AREA REQ'D = $\frac{58860+6720}{2700}$ = 24.3□'	

Figure D-1. Continued.

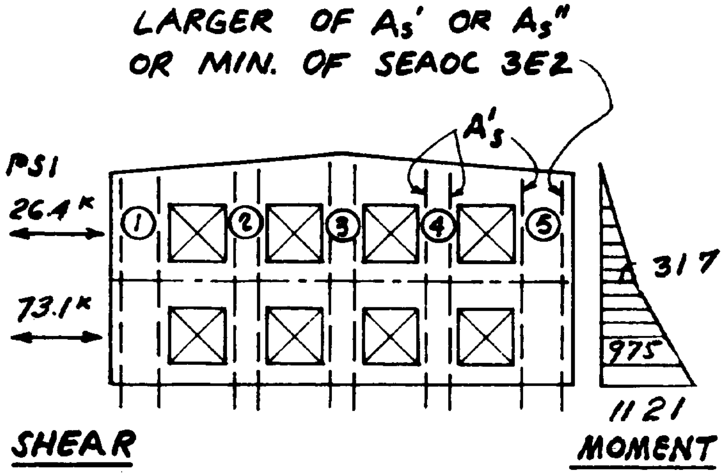
WALL DESIGN

WALL 1

$U = 1.4 (D+L+E)$

$U = 0.9D + 1.4E$

$v = 2\sqrt{f'_c} = 126$ PSI



THE INDIVIDUAL PIERS IN WALL 1 SHOULD BE DESIGNED FOR THE BENDING MOMENT DUE TO THE LATERAL LOAD ($M = V \times h/2$) PLUS THE AXIAL LOADS FROM DEAD AND LIVE LOADS (EXCEPT ROOF L.L.) PLUS THE AXIAL LOAD DUE TO WALL OVERTURNING. UNLESS AXIAL LOADS ARE EXCEPTIONALLY LARGE IT IS USUALLY CONSERVATIVE TO NEGLECT AXIAL LOADS. THIS PROBLEM PROCEEDS ON THIS SIMPLIFYING ASSUMPTION.

SEE NEXT SHT. FOR SAMPLE CALCULATION — FIRST STORY

	PIER	WIDTH	R (P. 9)	V	A_c	$v = \frac{1.4\,V}{\phi A_c}$	$M = V\frac{h}{2}$	A'_s	$1.33A'_s$	REINF.
1ST STORY	1	72"	11.1	22.7	720□"	74	68.1	0.46□"	0.61	2-#5
	2	48"	4.5	9.2	480	45	27.6	0.28	0.37	2-#5
	3	48"	4.5	9.2	480	45	27.6	0.28	0.37	2-#5
	4	48"	4.5	9.2	480	45	27.6	0.28	0.37	2-#5
	5	72"	11.1	22.7K	720	74	68.1	0.46	0.61	2-#5
			$\Sigma=35.7$	$\Sigma=73.1^K$ (P.15)		$\phi=0.6$	$h/2=3'$		NOTE 1	

NOTE: 1. A'_s INCREASED BY 1/3 PER ACI ¶ 10.5.2
2. MIN. REINF. 2-#5 PER FIG. 6-6
3. REINF. FOR 1ST STORY PIERS IS EXTENDED TO THE 2ND STORY

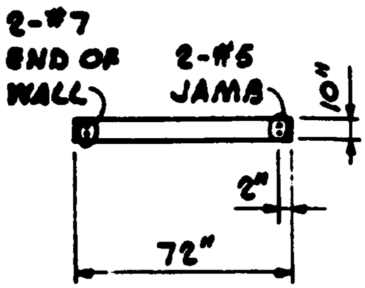
Figure D-1. Continued.

WALL DESIGN (CONT.)

WALL 1

SAMPLE CALCULATION FOR SHT. 21:

1ST STORY - PIER 1



PLAN - PIER 1

SHEAR IN PIER 1

$V_1 = \frac{R}{\Sigma R} \times V_{WALL} = \frac{11.1}{35.7} \times 73.1^{K} = 22.7^{K}$ (P.9) (P.15)

$V_u = 1.4 \times V_1 = 31.8^{K}$

$v = \frac{V_u}{\phi A_c} = \frac{31.8}{0.60 \times 720} = 74 \text{ PSI} < 126 \text{ PSI}$ OK

MOMENT IN PIER 1 DUE TO PIER SHEAR

$M = V_1 \times \frac{h}{2} = 22.7 \times \frac{6'}{2} = 68.1$

$M_u = 1.4M = 1.4 \times 68.1 = 95.3'^{K}$

REQ'D $A_s' = \frac{M_u}{\phi f_y (d - \frac{a}{2})} = \frac{95.3 \times 12}{0.9 \times 40 \times (70 - \frac{1.0}{2})} = 0.46 \,\square''$

WHERE a IS ASSUMED AS 1.0

CHECK ASSUMPTION:

$a = \frac{A_s f_y}{0.85 f_c' b_w} = \frac{0.40 \times 40}{.85 \times 4 \times 10} = 0.47 < 1.0$ ASSUMED

$\therefore$ OK

SEE SHT. 23 FOR BOUNDARY MEMBERS

Figure D-1. Continued.

WALL DESIGN

WALL 1 (CONT.)

BOUNDARY MEMBER FOR ENTIRE WALL

GRAVITY LOADS (P. 17)

$W_D = 3684\ \#/_1 \times 48' = 177^K$

$W_L = 800\ \#/_1 \times 48' = 38^K$ (2ND FLOOR ONLY)

OVERTURNING MOMENT (P.16)

$M_{OT} = 1121$

USE LOAD COMBINATION $0.9D - 1.4E$

LOAD AT EACH END =

$$F = C_D - T_M = \frac{0.9 W_D}{2} - \frac{1.4 M_{OT}}{0.9D}$$

$$= \frac{0.9(177)}{2} - \frac{1.4(1121)}{0.9(48')} = 79.7 - 36.3$$

SINCE $C_D > T_M$ THERE IS NO TENSION.

PROVIDE TRIM REINF. =

$$A_s'' = \frac{T_M}{\phi f_y} = \frac{36.3}{0.9 \times 40} = 1.01\ IN^2$$

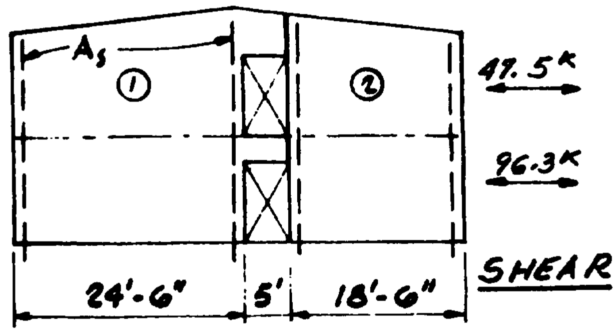
2 - #7 AT EACH END OF WALL

Figure D-1. Continued.

WALL DESIGN

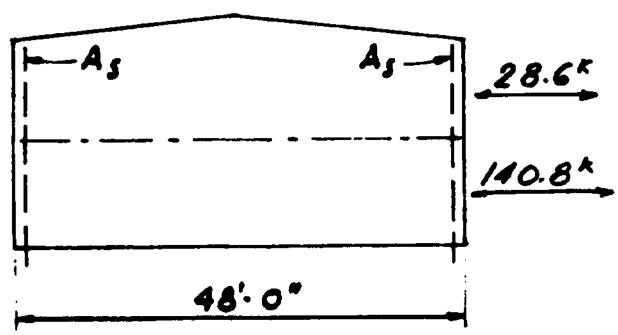
WALL 3 (WALL 5 SIM.)

ASSUME THAT PIERS ACT AS SERIES OF VERTICAL CANTILEVER BEAMS STRUTTED AT ROOF & 2ND FLR. LINE & FIXED AT 1ST FLR.



	PIER	WIDTH	R (P.9)	V	A_c	$v=\frac{1.4V}{\phi A_c}$	M_{OT}	A_s *	REINF
1ST STORY	1	24.5'	27	57.9^K	2940 □"	46 PSI	$980^{'k}$	2.07 □"	2-#9
	2	18.5'	17.9	38.4	2220	40	649	1.81	2-#9
			Σ=44.9	$\Sigma=96.3^K$ (P.16)		$\phi=0.6$	$\Sigma=1629^{'k}$ (P.16)		

WALL 7



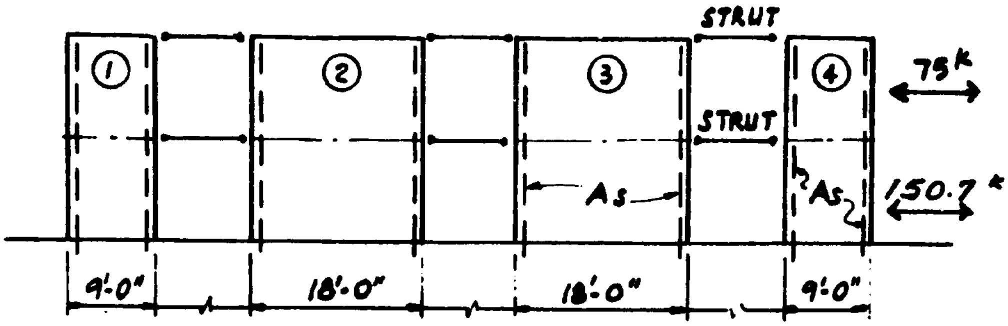
	PIER	WIDTH	R (P.10)	V	A_c	$v=\frac{1.4V}{\phi A_c}$	M_{OT}	A_s *	REINF
1ST STORY	1	48'	58.8	140.8^K	5760	57 PSI	1892 (P.16)	2.04	2-#9
						$\phi=0.6$			

* INCREASED BY 1/3 PER ACI ¶ 10.5.2

Figure D-1. Continued.

WALL DESIGN

WALL A (WALL C SIM.)



	PIER	WIDTH	R (p/10)	V	Ac	$\tau = \frac{1.4V}{\phi Ac}$	M_{OT}	As	
1ST. STORY	1	9'	4.5	19.0^k	1080	41 psi	314	1.82	2-#9
	2	18'	13.3	56.3	2160	61	928	2.67	3-#9
	3	18'	13.3	56.3	2160	61	928	2.67	3-#9
	4	9'	4.5	19.0	1080	41	314	1.82	2-#9
			Σ 35.6	$\Sigma\, 150.7^k$ (p/c)		$\phi = 0.6$	$\Sigma\, 2483'^k$ (p/k)		

SAMPLE CALCULATION FOR PIER 1 :

SHEAR $V = \frac{R}{\Sigma R} V_{WALL} = \frac{4.5}{35.6} \times 150.7 = 19.0^k$

$V_u = 1.4V = 1.4 \times 19.0^k = 26.6^k$

$\tau = \frac{1.4V}{\phi Ac} = \frac{V_u}{\phi Ac} = \frac{26.6}{0.6 \times 1080} = 41$ psi < 126 psi OK

MOMENT $M_1 = \frac{R}{\Sigma R} M_{OT} = \frac{4.5}{35.6} \times 2483'^k = 314$

$M_u = 1.4 M_1 = 1.4 \times 314 = 440'^k$

$A_s = \frac{M_u}{\phi f_y (d - \frac{a}{2})} = \frac{440 \times 12}{0.9 \times 40 \times (108" - \frac{3}{2})} = 1.37^{\square"}$

$A_s = 1.33 \times 1.38 = 1.82^{\square"}$ INCREASE PER ACI # 10.5.2

Figure D-1. Continued.

WALL DESIGN

WALL A – CONT. (WALL C SIM.)

BOUNDARY MEMBER – PIER 1

$M_u = 440^{K'}$ (P. 25)

$W_D = 29.4^K$, $W_L = 2.4^K$ (P. 20)

$C_D = \frac{0.9 W_D}{2} \quad \frac{0.9 \times 29.4}{2} = 13.2$

$T_M = \frac{M_u}{0.9d} = \frac{440}{0.9(9)} = 54.3$

$T = T_M - C_D = 54.3 - 13.2 = 41.1 = \phi A_s'' f_y$

$A_s'' = \frac{41.1}{0.9 \times 40} = 1.14 \text{ IN.}^2 < A_s$ P. 25

CHECK TRANSV. REINF. (ACI 21.5.3.1)

WALL $A = 10' \times 9/12 = 7.5 \text{ ft}^2$

$S = td^2/6 = \frac{10}{12} \; \frac{(9)^2}{6} = 11.2 \text{ ft}^3$

$M = M_u = 440^{K'}$

$P = 1.4 (W_D + W_L) = 1.4 (29.4 + 2.4) = 44.5^K$

$f = \frac{P}{A} + \frac{M}{S} = \frac{44.5}{7.5} + \frac{440}{11.2} = 45.2 \text{ ksf}$

OR 314 psi $< (0.2 f_c' = 800 \text{ psi})$

NO TRANSV. REINF. REQ'D

Figure D-1. Continued.

WALL DESIGN

WALL A-CONT. (WALL C SIM.)

BOUNDARY MEMBER - PIER 2

$M_h = 928, \quad M_u = 1.4 \times 928 = 1299$

$W_D = 58.9 \quad W_L = 4.8$

$C_D = \frac{0.9 W_D}{2} = \frac{0.9 \times 58.9}{2} = 26.5$

$T_M = \frac{M_u}{0.9d} = \frac{1299}{0.9 \times 18'} = 80.2$

$T = 80.2 - 26.5 = 53.7$

$A_s'' = \frac{53.7}{0.9 \times 40} = 1.49 < A_s$

CHECK TRANSV. REINF.

WALL $A = 18' \times \frac{10}{12} = 15 \text{ ft.}^2$

$S = \frac{10}{12} \times \frac{(18)^2}{6} = 45 \text{ ft.}^3$

$P = 1.4\ (58.9 + 4.8) = 89.2^k$

$M_u = 1299$

$f = \frac{89.2}{15} + \frac{1299}{45} = 6.0 + 28.9 = 34.9 \text{ ksf}$

OR 242 psi < 800

NO TRANSV. REINF. REQ'D

Figure D-1. Continued.

WALL DESIGN - SEISMIC FORCES NORMAL TO WALL

$$F_p = Z\,I\,C_p\,W_p$$

WHERE $C_p = 0.75$ (SEAOC TABLE 1-H)

$W_p = 125^{\#}/_{\square'}$ (10" CONC)

$Z = 0.4 \quad I = 1.0$

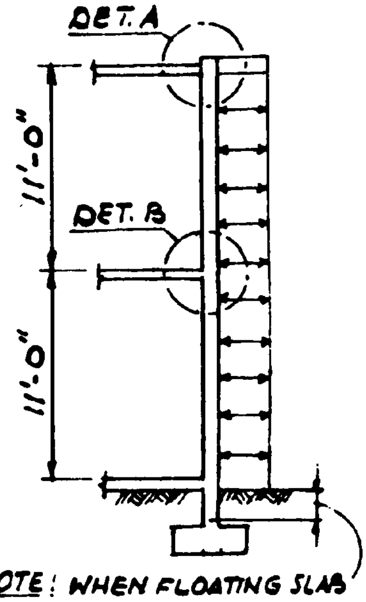
$F_p = F_p = 0.4 \times 1.0 \times 0.75 \times 125 = 37.5^{\#}/_{\square'}$

REACTION @ ROOF $= 37.5^{\#}/' \times 11/2 = 206^{\#}/'$

CONT. SPAN

REACTION @ 2ND FLR $= 37.5^{\#}/' \times 11' \times \frac{10}{8} = 516^{\#}/'$

MIM. LOAD $= 200^{\#}/'$ (SEAOC 1H2h)



NOTE: WHEN FLOATING SLAB IS USED, ASSUME PT. OF FIXITY 2' BELOW GRADE LINE FOR DESIGN OF WALL

MAX. WALL BENDING @ 2ND FLR LINE

16' TRIB TO LINES 1 OR 7

$$M = \frac{wl^2}{8} + \frac{M_{ecc}}{2}$$

$$= \frac{37.5^{\#} \times 11^2 \times 12}{8} + \frac{(73^{\#} + 50^{\#}) \times 16 \times 5'' ecc}{2}$$

$$= 6806 + 4920 = 11{,}726^{\#''}$$

ASSUME $a = 0.1''$

$$A_s = \frac{M}{\phi f_y (d - \frac{a}{2})} = \frac{11.73^{"K}}{0.9 \times 40 (8.5 - \frac{0.1}{2})}$$

$$= 0.039^{\square''}$$

CHECK ASSUMED $a = 0.1$

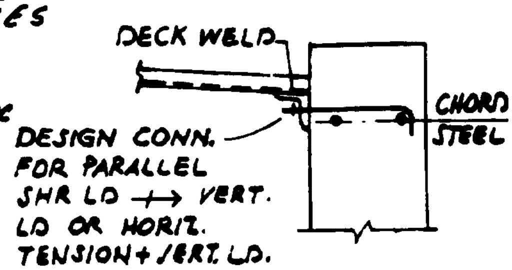
$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.039 \times 40}{0.85 \times 4 \times 10} = 0.05 < 0.1 \quad OK$$

MIN. $A_s = .0025\,bd$

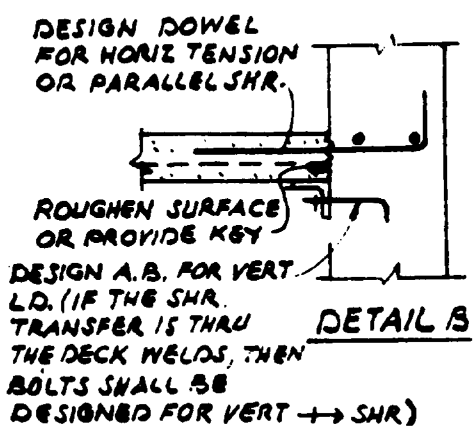
$= .0025 \times 10 \times 8.5 = 0.21^{\square''}$

USE #4 @ 16" O.C. E.F.

$A_s = 0.30$



DETAIL A

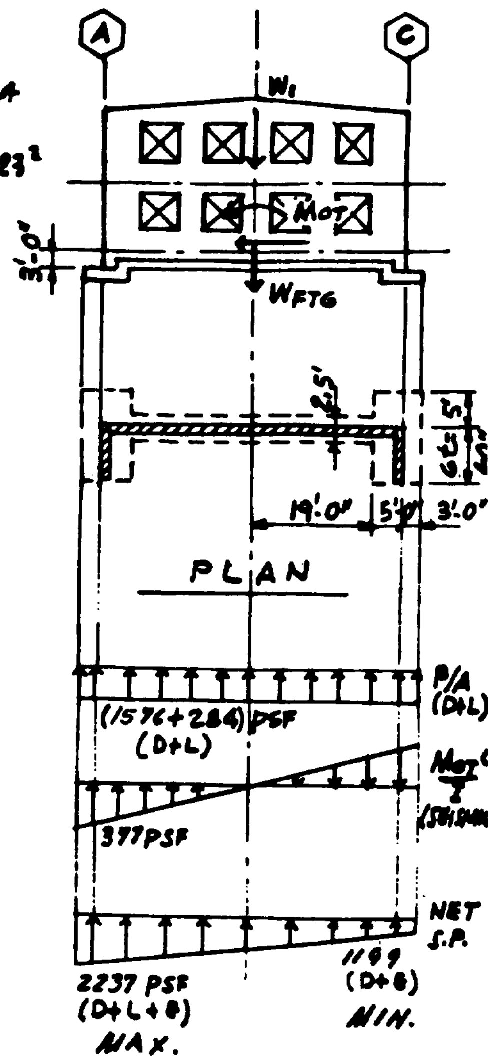


DETAIL B

Figure D-1. Continued.

FOOTING DESIGN FOR SEISMIC LOADS

WALL 1



MOMENT OF INERTIA OF FTG.:

FOR VERTICAL LOAD $I_1 = 2.5' \times \frac{48^3}{12} = 23040\ FT^4$

FOR SEISMIC LOAD $I_2 = \frac{2.5' \times 38^3}{12} + 2 \times 8' \times 10' \times 23^2$
INCLUDE RETURN WALL FTGS. $= 96072\ FT^4$

AREA OF FTG. = 2.5' × 54' = 135 □'

WEIGHT (p. 17)

$W_1 = (3684\ ^{\#}/_{'}) \times 48' = 176\,832\ ^{\#}$ (DEAD)

$W_1 = (800\ ^{\#}/_{'}) \times 48' = 38400^{\#}$ (LIVE w/o ROOF LL.)

$W_{FTG} = (751\ ^{\#}/_{'}) \times 48' = 36000$

ΣW (DEAD) = 212 832
ΣW (LIVE) = 38400

OVERTURNING MOMENT @ BASE OF FTG.

$M_{OT} = \underset{\text{℄(p.21)}}{1121'^{K}} + \underset{\text{℄(p.21)}}{73.7^{K}} \times 3' = 1340'^{K}$

SOIL PRESSURE		MAX.	MIN.
P/A (DEAD)	$\frac{212\,800}{135}$	+ 1576	+ 1576
P/A (LIVE)	$\frac{38400}{135}$	+ 284	
$M_{OT}C$	$\frac{1340 \times 27'}{96072}$	+ 377	− 377
		2237 < 4000	1199 NO UPLIFT

NOTE: THE SOIL PRESSURE UNDER THE RETURN WALLS DUE TO OVERTURNING (377 PSF) IS ADDED TO THE SOIL PRESSURE UNDER THE RETURN WALL WHICH IS VERY LOW ∴ OK

Figure D-1. Continued

FOOTING DESIGN FOR SEISMIC LOADS

WALL 7

MOMENT OF INERTIA OF FTG:

FOR VERTICAL Ld $I_1 = \frac{2.75 \times 48^3}{12} = 25344\ FT^4$

FOR SEISMIC Ld $I_2 = \frac{2.75 \times 38^3}{12} + 2 \times 8' \times 10' \times 23^2$

$= 97215\ FT^4$

AREA OF FTG = 2.75' × 54' = 148.5□'

WEIGHT (p.18)

W_1 = (4434 #/' × 48' = 212 832# (DEAD)

W_1 = (800 #/' × 48' = 38400 (LIVE W/o ROOF LL)

W_{FTG} = (806 #/' × 48' = 38736

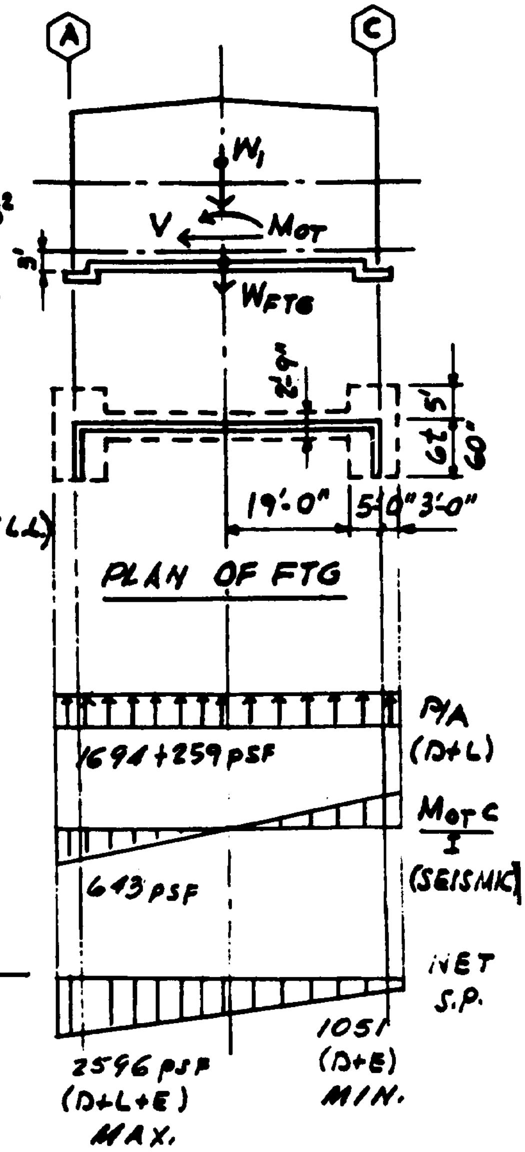
ΣW (DEAD) = 251 568#

ΣW (LIVE) = 38400

OVERTURNING MOMENT AT BASE OF FTG.

(p.16)

$M_{OT} = 1892 + 140.8^k \times 3' = 2314'^k$



SOIL PRESSURE		MAX	MIN.
P/A (DEAD)	$\frac{160788}{148.5}$	+ 1694	+ 1694
P/A (LIVE)	$\frac{38400}{148.5}$	+ 259	
$\frac{M_{OT} C}{I_2}$	$\frac{2314 \times 27}{86212}$	+ 643	- 643
		+ 2596	+ 1051
		< 4000	NO UPLIFT

Figure D-1. Continued.

FOOTING DESIGN FOR SEISMIC LOADS

WALL 3 (WALL 5 SIM.)

MOMENT OF INERTIA OF FTG:

FOR VERTICAL LD $I_1 = \frac{3 \times 48^3}{12} = 27648 \text{ FT}^4$

FOR SEISMIC LD $I_2 = \frac{3 \times 38^3}{12} + 2 \times 8' \times 7.5 \times 23^2$

$= 77200 \text{ FT}^4$

AREA OF FTG. $3' \times 54' = 162^{\square'}$

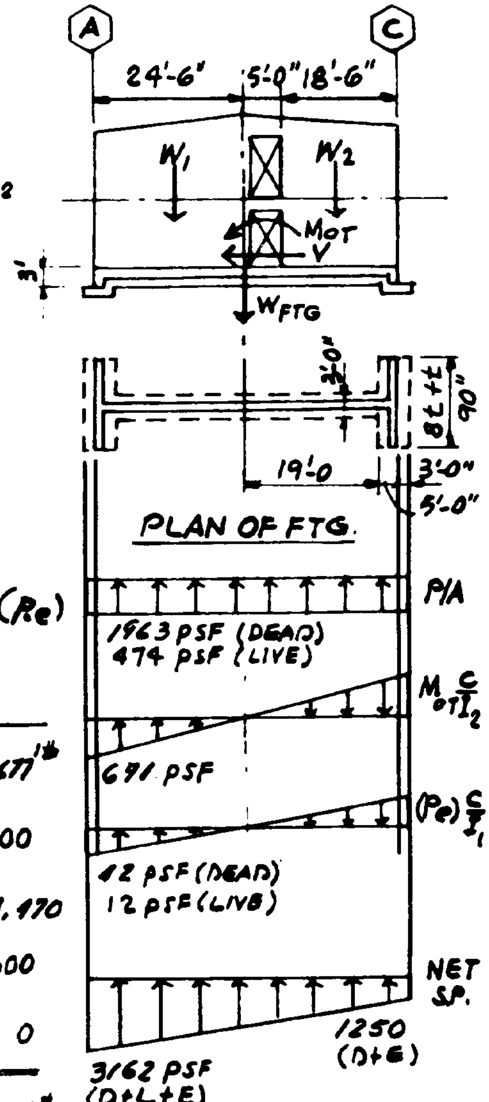
OVERTURN'G MOMENT $= 1629^{'K}$ (p.16)

SHEAR V $= 96.3^K$ (p.16)

OVERTURN'G MOMENT @ BASE OF FTG.

$M_{OT} = 1629^{'K} + 96.3^K \times 3 = 1918^{'K}$

CALCULATION OF ECCENTRIC MOMENT (P_e) OF WALL MASS RESPECT TO N.A.



WEIGHTS (p.19)		X DIST. TO N.A. FTG	= WX
W_1 (DEAD) $= 5761$ #/' $(24.5' + 2.5')$	$= 155\,547$#	x -11.75	$= -1,827,677$ '#
W_1 (LIVE) $= 1600$ #/' $(24.5' + 2.5')$	$= 43200$	x -11.75	$= -507600$
W_2 (DEAD) $= 5761$ #/' $(18.5' + 2.5')$	$= 120\,981$	x $+14.75$	$= 1,784,470$
W_2 (LIVE) $= 1600$ #/' $(18.5' + 2.5')$	$= 33600$	x $+14.75$	$= 495600$
$W_{FTG} = 863$ #/' $\times 48'$	41400	x 0	$= 0$

ΣW (DEAD) $= 317,928$# ΣW(DEAD) $- 43\,207$#

ΣW (LIVE) $= 76800$# ΣW(LIVE) $- 12000$#

THESE SOIL PRESSURES ARE CALCULATED ON THE NEXT PAGE

ECCENTRICITY $e = \frac{-43207^{\#'}}{317928^{\#}} = -0.14'$ (DEAD)

$e = \frac{-12000}{76800} = -0.156'$ (LIVE)

NOTE: THESE ARE SMALL, THE EFFECTS COULD BE NEGLECTED.

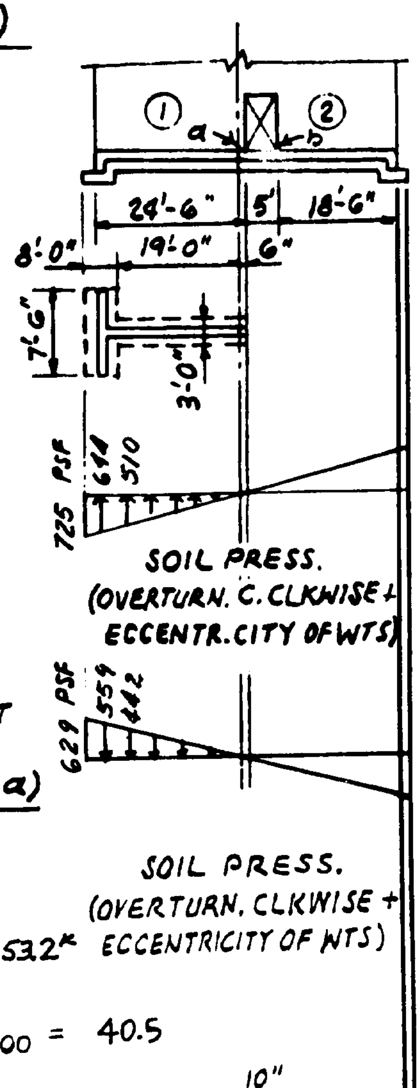
Figure D-1. Continued.

FOOTING DESIGN FOR SEISMIC LOADS

WALL 3 (CONT.) (WALL 5 SIM)

SOIL PRESS. PSF		MAX.	MIN.
$\frac{P}{A}$ (DEAD)	$\frac{317,928}{162}$	+1963	+ 1963
$\frac{P}{A}$ (LIVE)	$\frac{76800}{162}$	+ 474	
$M_{OT}\ \frac{C_2}{I_2}$	$\frac{1918 \times 27}{77200}$	+ 671	− 671
$M_{ecc}\ \frac{C_1}{I_1}$ (DEAD)	$\frac{43207 \times 27}{27648}$	+ 42	− 42
$M_{ecc}\ \frac{C_1}{I_1}$ (LIVE)	$\frac{12000 \times 27}{27648}$	+ 12	

$\frac{3162}{4000}$ < + 1250 NO UPLIFT



CHECK SHEAR IN FDN. WALL @ DOOR OP'N'G (Pt a)

DL + LL $W = 5' \times 3'\ (1.963 + 0.474) = 36.6^{K}$

$V = 36.6/2 = 18.3^{K}$

SEISMIC (CCW OVERTURNING)

$V = (510 \times \frac{19'}{2} \times 3') + (644 \times 8' \times 7.5') = 53.2^{K}$

$V_u/\varphi = \frac{1}{0.60}(18.3 + 53.2) = 119^{K}$

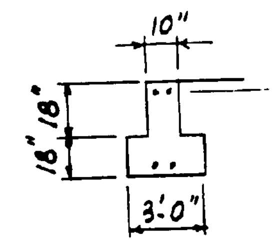
$V_c = 2\sqrt{f_c'}\ bd = 2\sqrt{4000} \times 10 \times 32/1000 = 40.5$

$V_s = 119 - 40.5 = 78.5$

$V_s < 8\sqrt{f_c'}\ bd = 162$ OK

PROVIDE # 4 ⊓ @ 6" OK

$V_s = \frac{0.40 \times 40 \times 32}{6} = 85.3 > 78.5$



SECTION

Figure D-1. Continued.

FOOTING DESIGN FOR SEISMIC LOADS

WALL 3 (CONT.)

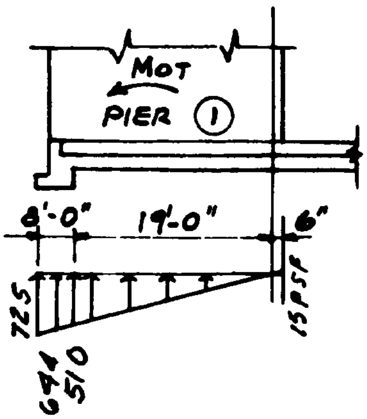
CHECK MOMENT IN FDN WALL @ DOOR OP'N'G (Pt. a)

$$M_{D+L} = \frac{WL}{10} = \frac{36.6 \times 5}{10} = 18.3^{'K}$$

$$M_{OT}(\text{PIER } 1) = \Sigma M_{OT}$$

$$= \frac{27}{44.9} \times 1918^{'K} = 1153^{'K}$$

RIGIDITIES, P.24



MOMENT AT Pt. a (SEISMIC)

$$M_a = 1153^{'K} - \left[(510 \times \frac{19}{2} \times 3'\text{WIDE}) \times (19' \times \frac{2}{3} + 0.5')\right] - \left[644 \times 7.5' \times 8' \times 23'\right] = 72.9^{K}$$

$$M_u = 1.4(18.3 + 72.9) = 128^{'K}$$

$$F = \frac{bd^2}{12000} = \frac{10 \times 32^2}{12000} = 0.86$$

$$K = \frac{M_u}{F} = \frac{128}{0.86} = 149$$

$$a_u = 2.92 \quad (\text{ACI} \cdot \text{SP.17 FLEX 1-2})$$

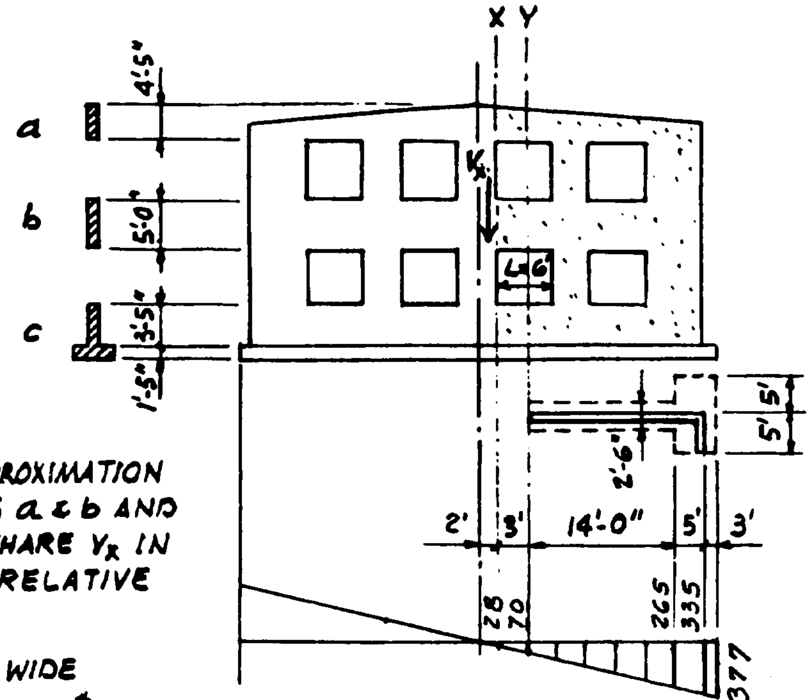
$$A_s = \frac{M_u}{a_u d} = \frac{128}{2.96 \times 32} = 1.40^{\square''}$$

2-#8 TOP & BOTT

Figure D-1. Continued.

SPANDREL DESIGN

WALL 1



AS A SIMPLYFYING APPROXIMATION ASSUME THAT SPANDRELS a & b AND FOUNDATION WALL C SHARE V_X IN PROPORTION TO THEIR RELATIVE RIGIDITIES

$V_X = \frac{265+28}{2} \times 17' \times 2.5$ WIDE

$+335 \times 10' \times 8' = 33000^{\#}$ SHEAR DUE TO SEISMIC OVERTURNING

SOIL PRESSURE (SEISMIC)
(p.29)

$V_y = \frac{265+70}{2} \times 14 \times 2.5 + 335 \times 8 \times 70 = 32700^{\#}$

	$I(FT^4)$	L	$R=I/L$	$R/\Sigma R$	V_x	$v=\frac{V}{\phi A_c}$	V_y	$M=\frac{VL}{2}$	W_{D+L}	$M'=\frac{wl^2}{12}$	M+M'
4.5'	6.3	6'	1.05	0.22	7.3^K	22 psi	7.2	21.6 'K	954#/'	2.86'K	24.5 'K
5'	8.68	6'	1.45	0.31	10.2	28	10.1	30.3	2593#/'	7.78'K	38.1 'K
5' / 2.5'	13.31	6'	2.22	0.47	15.5	43	15.4	46.2	1178#/'	3.5'K	49.7 'K
			4.72	1.0	33.0^K		32.7^K				

DESIGN SPANDREL FOR MAX. MOMENT (M+M')

SAMPLE CALCULATION: $V_X = 33.0^K \times R/\Sigma R = 33.0^K \times .22 = 7.3^K$

$W_{D+L} = (391^{\#/'}) + 4.5' \times 125^{\#} = 954^{\#/'}$ UNIT WT. ON SPANDREL

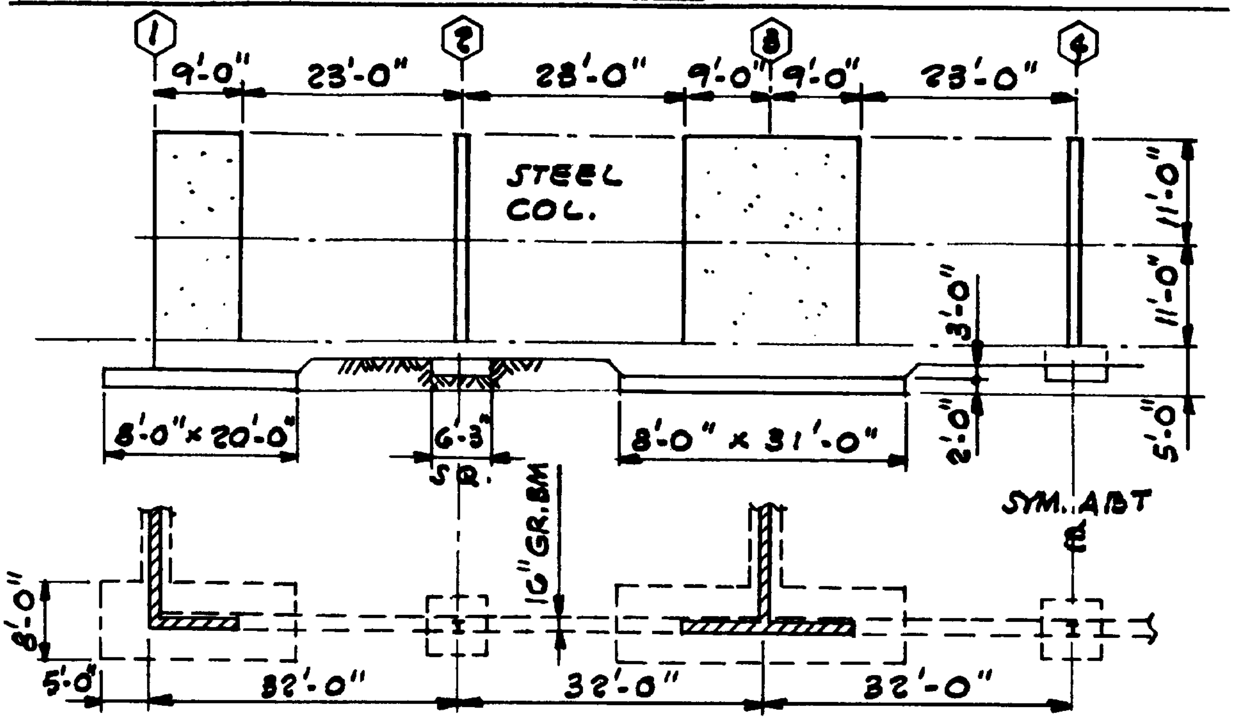
℄(p.18)

NOTE: ROOF LL NEGLECTED WHEN COMBINED WITH SEISMIC (SEAOC SEC. 1H1a.)

Figure D-1. Continued.

FOOTING DESIGN FOR SEISMIC LOADS

WALL A (WALL C SIM.)



PLAN OF FOOTINGS

Figure D-1. Continued.